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The Effect of Scale on the Mechanical Properties of Jointed Rock Masses

Francois E. Heuze

DTRA Advanced Schoolhouse Course, Springfield, VA, June 14-15, 2004

U.S. Department of Energy

Lawrence Livermore National Laboratory

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ABSTRACT

These notes were prepared for presentation at the Defense Threat Reduction Agency's (DTRA) Hard Target Research and Analysis Center (HTRAC), at the occasion of a short course held on June 14-15, 2004.

The material is intended for analysts who must evaluate the geo-mechanical characteristics of sites of interest, in order to provide appropriate input to calculations of ground shock effects on underground facilities in rock masses. These analysts are associated with the Interagency Geotechnical Assessment Team (IGAT).

Because geological discontinuities introduce scale effects on the mechanical properties of rock formations, these large-scale properties cannot be estimated on the basis of tests on small cores.

Accordingly, the outline of the lecture is as follows:

- Geological discontinuities
  - effect on ground shock
  - effect on failure of underground structures
  - basic mechanical properties
  - scale effects
- Rock Masses
  - deformability of rock masses
  - scale effects
  - strength of rock masses
  - scale effects
  - geological strength index (GSI)
- References

THE EFFECT OF SCALE ON THE MECHANICAL PROPERTIES OF JOINTED ROCK MASSES

Francois E. Heuze
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DTRA Advanced Schoolhouse Course

Springfield, VA

June 14-15, 2004

Outline

Geological discontinuities

- effect on ground shock
- effect on failure of underground structures
- basic mechanical properties
- scale effects

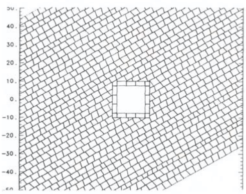
Rock Masses

- deformability of rock masses
- scale effects
- strength of rock masses
- scale effects
- geological strength index (GSI)

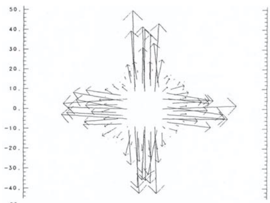
References

Geological discontinuities

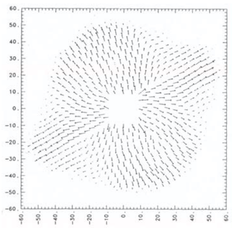
Effects of rock discontinuities - ground shock



Configuration



Early time velocity field

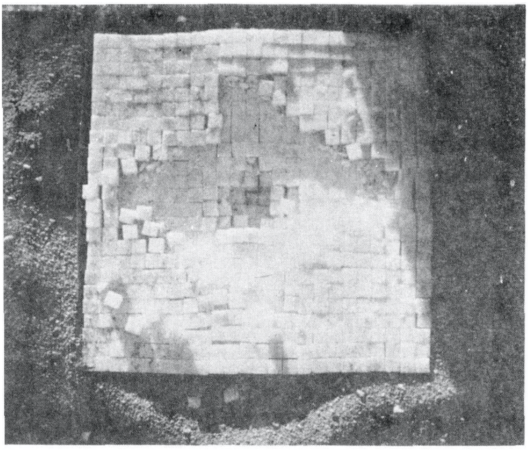


Later time velocities

After Walton et al, 1991.

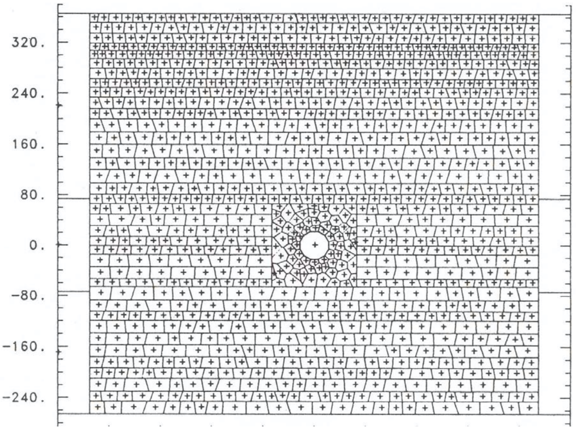
This ground shock pattern had been observed in "sugar cube" tests by Melzer (1970).

Effects of rock discontinuities - ground shock



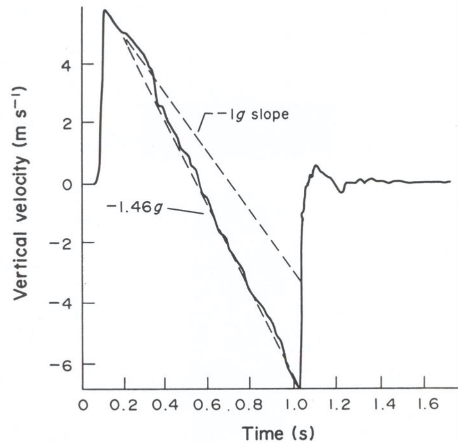
After Melzer (1970). Courtesy of S. Blouin.

Effects of rock discontinuities - ground shock



Modeling of a SHOAL-like event (12kt) with the DIBS discrete element code (after Heuze et al, 1993).

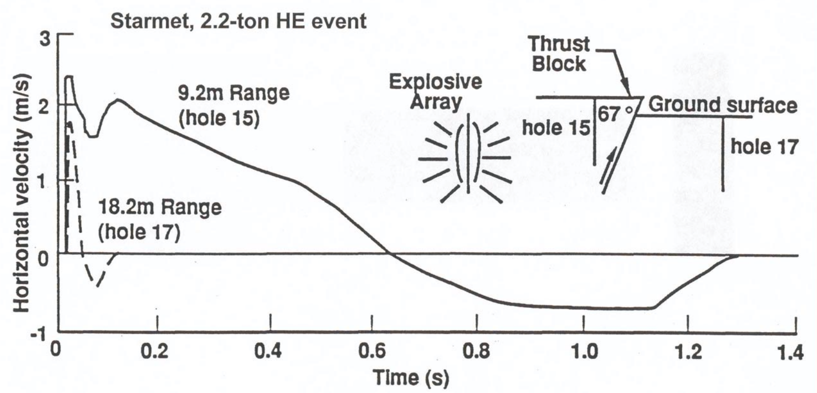
Effects of rock discontinuities - ground shock



(after Heuze et al, 1993)

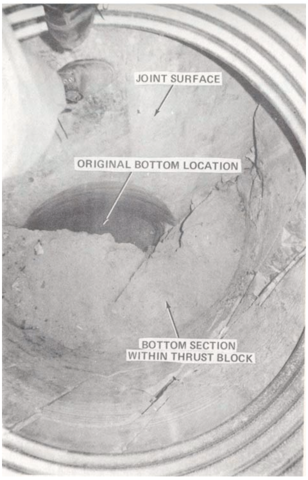
The DIBS modeling of a SHOAL-like event showed for the first time a surface spall return acceleration well in excess of 1g, as had been observed in SHOAL.

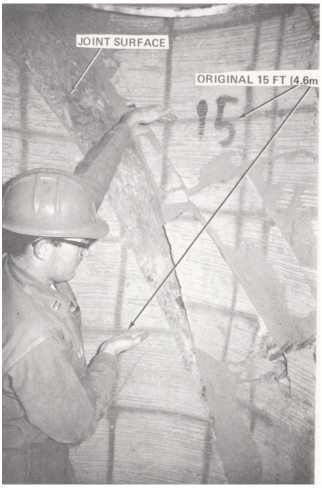
Effects of rock discontinuities - ground shock



STARMET HE event in granite, NM (Blouin and Kaiser, 1972). Note the very large influence of a geologic discontinuity on the displacement field.

Effects of rock discontinuities - ground shock





Model missile silos in the STARMET event (Blouin and Kaiser, 1972)

Effects of rock discontinuities - ground shock

Tunnel in tuff, Nevada Test Site

Effects of rock discontinuities - ground shock

Tunnel in tuff,
Nevada Test Site

Effects of rock discontinuities - slopes

In granite, near
Tioga Pass, CA

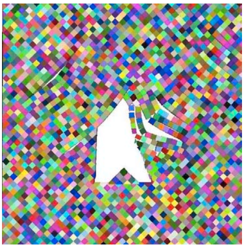
Effects of rock discontinuities - slopes

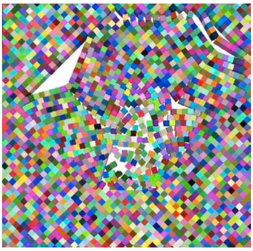
Near Libby Dam, MT (courtesy D. Lachel)

Effects of rock discontinuities - coal mines

Ground failure in a Belgian coal mine after a coal bump

Tunnel failure kinematics

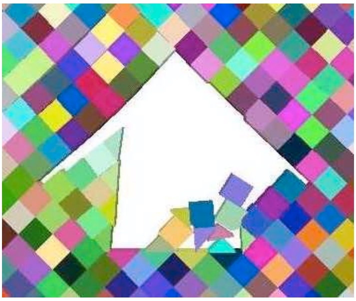




Simulations that discretely include geological discontinuities are required to model the mechanics of failure of tunnels in jointed rocks. Generic discrete element calculations are shown for illustration (Heuze, 2004).

Tunnel failure kinematics (cont.)

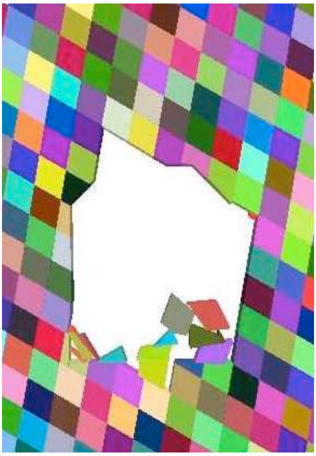
Damaged gold mine entry after a rock burst



Generic LDEC calculation (Heuze, 2004)

Tunnel failure kinematics

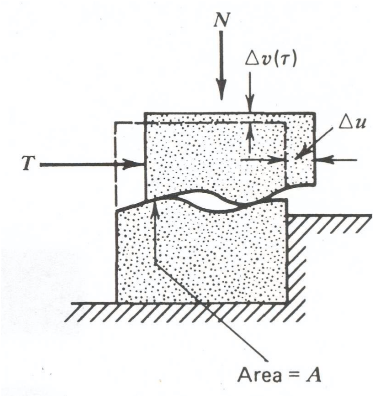
Damaged gold mine entry after a rockburst



Generic LDEC calculation (Heuze, 2004)

Mechanical attributes

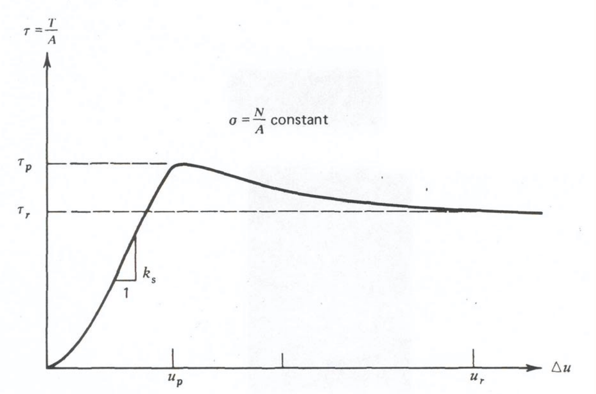
The basic mechanical concept of a "joint":



- **Normal stress : σ = N/A**
- **Shear stress τ =T/A**
- **Shear displacement : u or Δu**
- **Normal displacement : v or Δv**

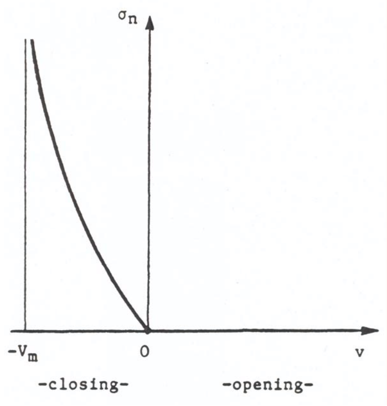
- **The mechanical properties of interest under shear stress and normal stress conditions are the stiffnesses (shear and normal), the dilatancy, and the shear strength (Goodman, 1980).**

Behavior of a joint under shear under constant σ



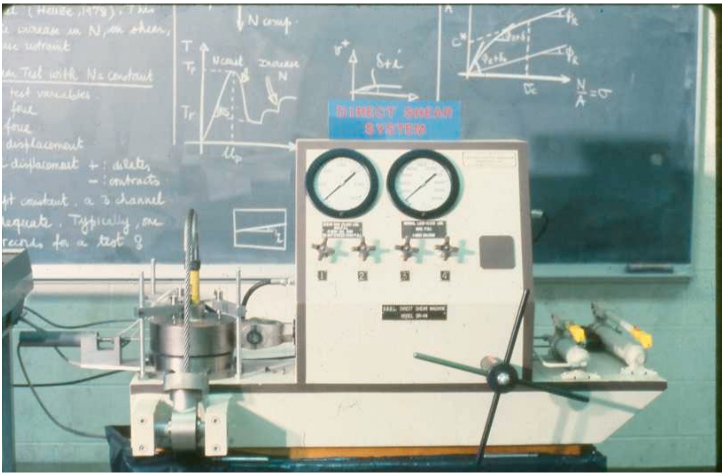
The shear stiffness is Ks. The peak shear strength is τ_p and the residual shear strength is τ_r. Rough joints also can dilate during shearing (Goodman, 1980).

Behavior of a joint under compression



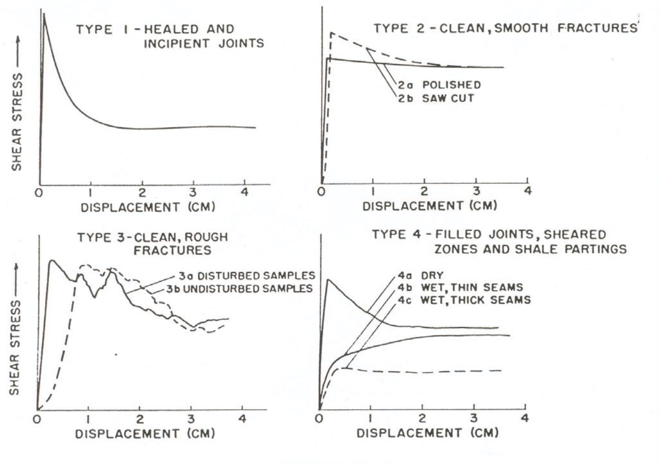
The normal stiffness, Kn, is the slope of the s,v curve and the joint has a maximum closure v_m. Kn increases as the joint closes (Goodman, 1980).

Shear testing machines with control of normal stress



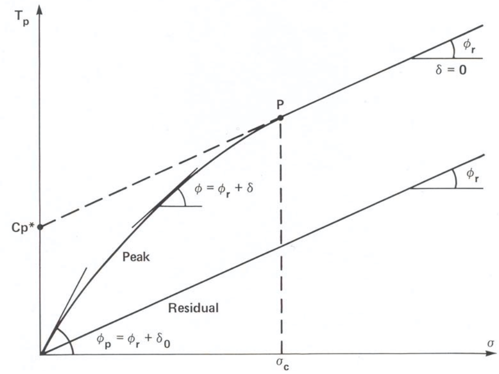
System at C.U. Boulder (1979), after a design by SBEL, Phoenix, AZ

Typical shear stress-deformation behavior



After Goodman, 1970

Shear strength envelope of joints under constant σ



In the τ-σ plane there are two envelopes: one for peak shear strength and one for residual shear strength. Above $\sigma = \sigma_c$ there is no dilation in shear.

A perspective on shear strength (N. Barton)

Barton,s (1973) empirical equation for peak shear strength:

$$\tau_p = \sigma_n \tan[\ JRC \log_{10} (JCS/\sigma_n) + \phi_r\ ]$$

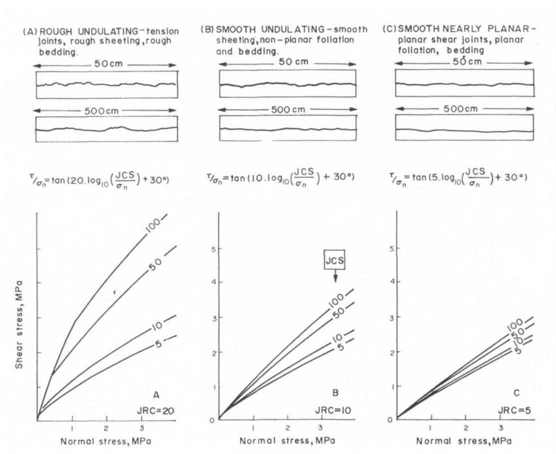
σ_n : normal stress on the joint

JCS : effective joint wall compressive strength (often taken as σ_c)

σ_c : wall rock unconfined compressive strength

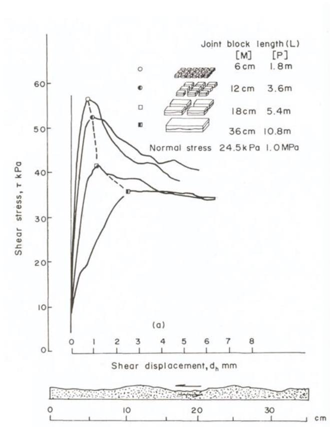
JRC : joint roughness coefficient

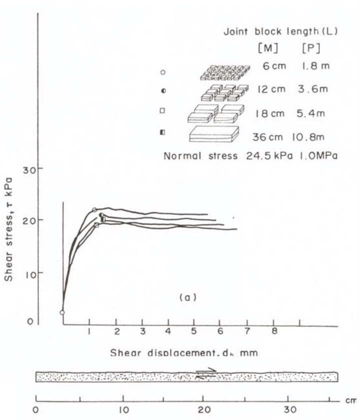
A perspective on shear strength (cont.)



Examples of JRC values and shear strength for different JCS values (Bandis, Lumsden , and Barton, 1981)

Scale effects on shear strength (Bandis et al, 1981)





Experimental results

Rough joint: scale effect **Smooth joint: no scale effect**

Scale effects on joint shear strength (cont.)

Scaling equations proposed by Barton et al, 1985. The subscript n refers to in-situ. The subscript 0 refers to laboratory.

- **Shear displacement to peak shear strength. L is the sample dimension in meters.**

$$\delta\,(\mathrm{peak}) = \frac{L_n}{500}\left[\frac{\mathrm{JRC_N}}{L_n}\right]^{0.33}$$

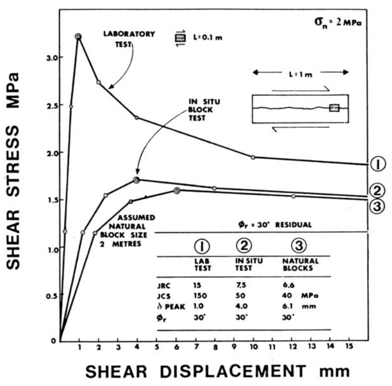
- **Joint Roughness Coefficient**

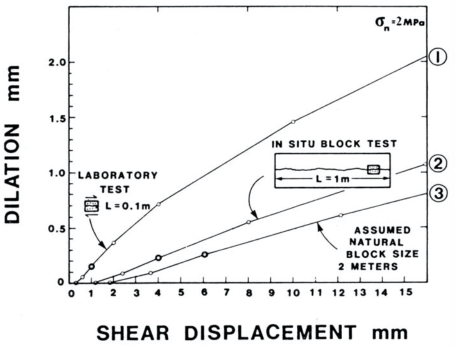
$$\mathrm{JRC_n} = \mathrm{JRC_0}\left[\frac{L_n}{L_0}\right]^{-0.02\mathrm{JRC_0}}$$

- **Joint Compressive Strength**

$$\mathrm{JCS_n} = \mathrm{JCS_0}\left[\frac{L_n}{L_0}\right]^{-0.03\mathrm{JRC_0}}$$

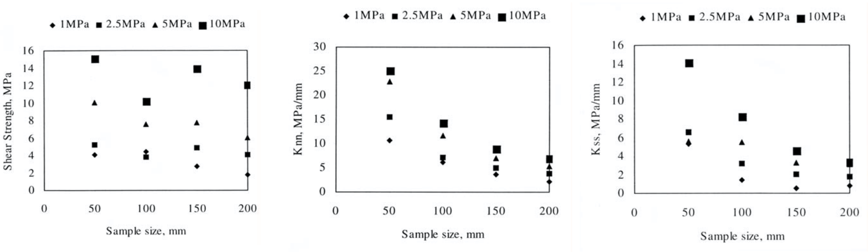
Scale effects on shear strength (cont.)





Laboratory results vs. expected in-situ results, based on the preceding scaling equations (Barton et al, 1985)

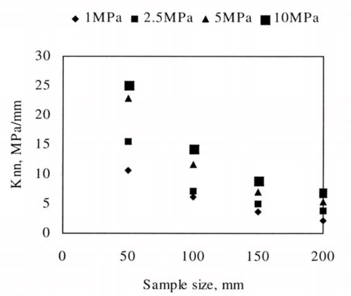
Scale effects on shear strength, and joint stiffnesses



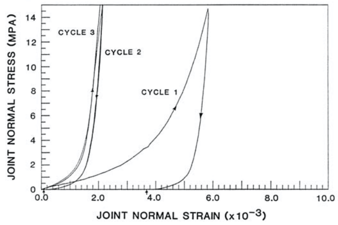
Tests on high-strength concrete replicas of a natural joint in granite, at various sizes and different constant normal stresses .

After Fardin et al., 2003.

Scale effects on normal stiffness; an explanation



For this figure, Fardin et al. indicate that Knn was calculated "at the linear part of the third loading cycle".



To explain this procedure, one can look at the results of cyclic normal loading of a sandstone joint reported by J. A. Brown et al. (LANL report BMO-TR-88-06, 1988).

Deformability of Rock Masses

Plate tests - Example (Wallace et al, 1970)

Note: USBR cost, 10 years ago, at Monk Hollow dam site, Utah, was 300K for 6 tests, not including rock surface preparation (G. Scott, pers. communic., 05/08/03)

Plate test analysis (Belin, 1959)

In isotropic media, the modulus of the rock mass is calculated as:

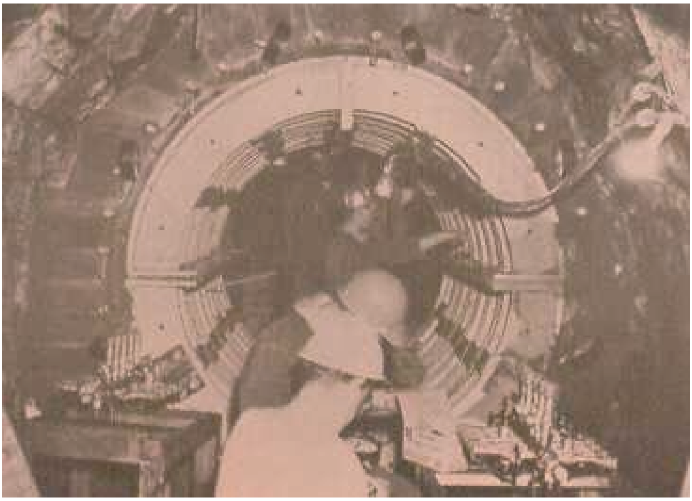
$$E = K \cdot P \cdot \pi \cdot a \cdot (1-\nu^2) / U$$

where

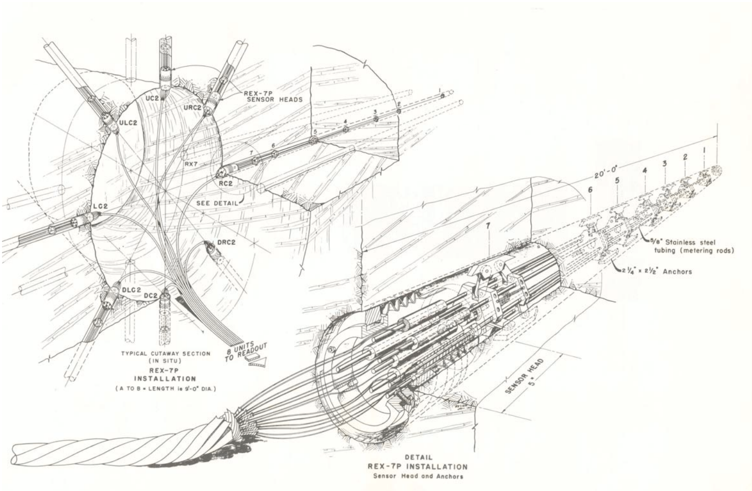
- K : coefficient = 0.50 for a perfectly rigid plate
  = 0.54 for a perfectly flexible plate
- P : applied pressure on the plate
- a : radius of the plate (assumed circular)
- ν : Poisson,s ratio of the rock mass (assume it to be 0.25)
- U : average displacement of the plate

Pressure chamber tests (Wallace et al, 1970)

Pressure chamber tests (Wallace et al, 1970)



Pressure chamber tests (Wallace et al, 1970)



Analysis of pressure tests in circular openings

This applies to tunnel tests such as above, or to dilatometer tests in boreholes.

Measuring the change in diameter, isotropic case:

$$E = \Delta P.\ D.(1+\nu)/\Delta D$$

where:

- ΔP : **increase in applied pressure**
- D : **diameter**
- ν : **Poisson,s ratio of the rock mass (assume 0.25)**
- ΔD : **change in diameter**

or

Measuring the displacement U(r) at depth "r" into the rock mass:

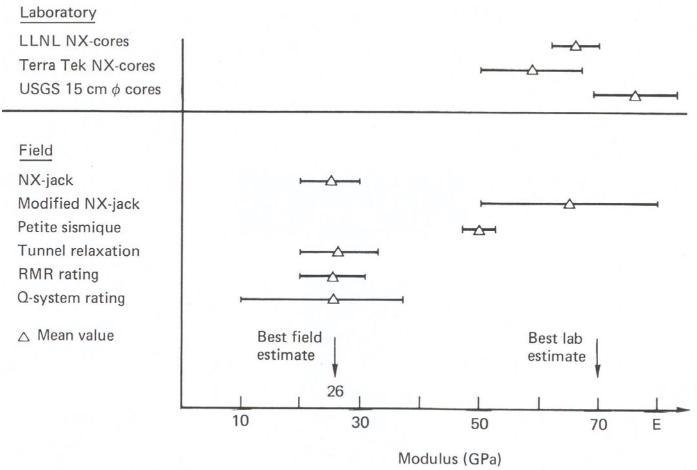
$$E = [\Delta P\ .\ D^2.\ (1+\nu)]/\ [4.\ r.\ U(r)]$$

The NX-Borehole Jack

See Goodman et al (1972), and Heuze and Amadei (1985).

Other field deformability tests - Flat jacks

Comparison of different tests - Scale effects



Climax granite, NTS, Nevada, (Heuze et al, 1982)

Static vs. dynamic moduli; ex: sedimentary rocks

Dam Name	Country	Rock	Area	E_{seism} 10^3kg/cm²	Test	P kg/cm²	E_{stat} (During Unl.) 10^3kg/cm²	$\frac{E_{seism}}{E_{stat}}$ (= :)
Sylvenstein	Germany	dolomite	right slope left slope	850 1100	jack load	40–160	71–146	12–5.8
Limberg	Austria	lam. limest.	slopes gallery	210–536 302–582	jack load	6–26	40–150	5.2–3.6
Speccheri	Italy	limestone	both slopes	550	jack load		600	1
Pieve di Cadore	Italy	limestone	right slope Pian delle Ere left slope	465 250–515 210	hydr. chamb.	12	35, inj. 52	10–7
Val Gallina	Italy	limestone	right slope left slope	185 175	hydr. chamb.		50–25, inj. 40 39	7.4–3.7 4.5
Vajont	Italy	limestone	upper slopes lower slopes	330–460 314–1400	hydr. chamb.	24 40	40–50 120	9–8 up to 11
Maë	Italy	limestone	valley bottom right slope left slope	870 310 260	hydr. chamb.	20	85, inj. 65, inj.	3.7 4
Fedaia	Italy	limestone	right slope left slope	385 395	hydr. chamb.	25	75, inj.	5.2

The moduli calculated from dynamic tests are generally much higher than those calculated from static tests. In seismic tests, the stress level is usually much lower than in static tests (after Link, 1964).

Static vs. dynamic moduli; sedimentary rocks (cont.)

		E(GPa)			G(GPa)			ν		
Well	Depth (m)	Lab.[a] Static	Lab.[a] Dyn.	Field Dyn.	Lab.[b] Static	Lab.[a] Dyn.	Field Dyn.	Lab.[a] Static	Lab.[a] Dyn.	Field Dyn.
PTS 24-19	1581.6	10.38	45.09	43.62	3.8	23.0	16.66	0.34	0.05	0.31
PTS 22-12	1958.0	16.99	49.50	29.98	6.57	25.12	11.08	0.29	0.024	0.35
PTS 3-10A	3512.5	41.66	66.02	51.12	17.18	33.04	21.04	0.21	0.008	0.21
RR 1-3	3803.6	22.59	63.61	45.21	9.23	36.52	18.31	0.26	0.15	0.24

[a]Values taken to be the average of E_x, E_z, G_{xy}, G_{xz}, or ν_{xy}, ν_{xz}.
[b]Values of laboratory static equals to G_{xy} (assuming $G_{xy} \simeq G_{xz}$).

3-way comparison of elastic constants for the Mesaverde sandstone (After Lin and Heuze, 1987)

Estimating joint normal stiffness in Climax granite

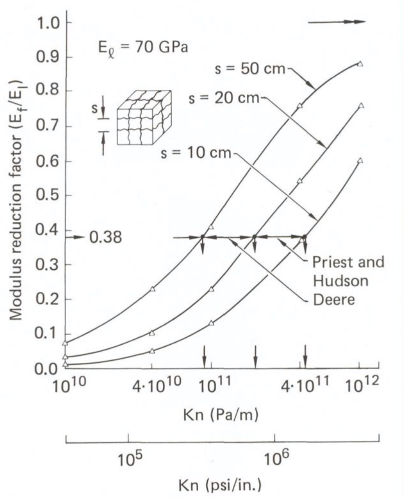
For a rock mass with three orthogonal joint sets, equally spaced, the field modulus is given by (Duncan and Goodman, 1968):

$$1/E_f = 1/E_r + 1/s.K_n$$

where

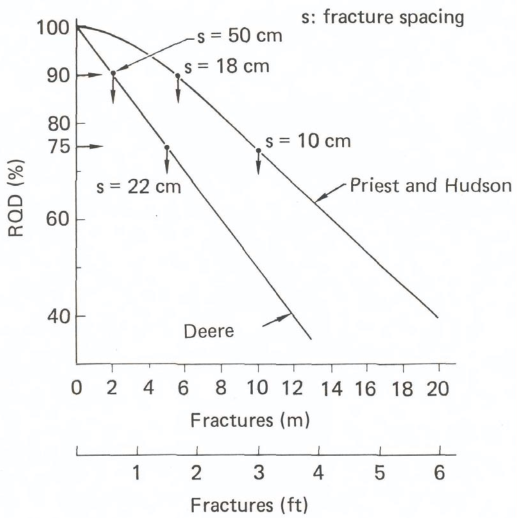
- **E_r = rock material modulus**
- **s = joint spacing**
- **K_n = normal joint stiffness**

The joint spacing could be estimated from the RQD (next slide).

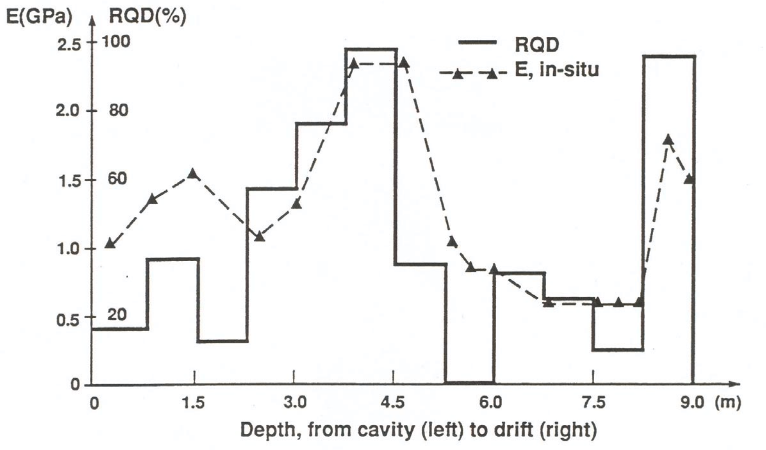


Joint spacing versus RQD

After Deere (1964), and Priest and Hudson (1976)



Rock mass modulus versus RQD



Example in tuff , Nevada Test Site, (Heuze et al., 1995)

Additional models of jointed rock masses

$$E_1 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_1 K_{n1}}\right)} \qquad G_{12} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_1 K_{s1}} + \frac{1}{S_2 K_{s2}}\right)} \qquad \nu_{12} = \nu_{13} = \nu_r \frac{E_1}{E_r}$$

$$E_2 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_2 K_{n2}}\right)} \qquad G_{13} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_1 K_{s1}} + \frac{1}{S_3 K_{s3}}\right)} \qquad \nu_{23} = \nu_{21} = \nu_r \frac{E_2}{E_r}$$

$$E_3 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_3 K_{n3}}\right)} \qquad G_{23} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_2 K_{s2}} + \frac{1}{S_3 K_{s3}}\right)} \qquad \nu_{31} = \nu_{32} = \nu_r \frac{E_3}{E_r}$$

E's : Young's moduli; G's : shear moduli; ν's ; Poisson's ratios

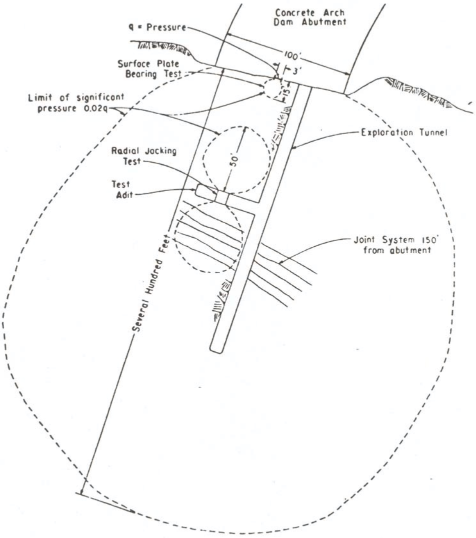
Three orthogonal joint sets, not equally spaced (Duncan and Goodman, 1968).

See also Gerrard (1982), and Fossum (1985)

Comparison of different tests - Scale effects

(Wallace et al, 1972)

Different tests will exercise different volumes of the rock mass at different stress levels.



Comparison of different tests - Scale effects

Type of test	"Test Volume" dm³	ft³
Strength		
81 cm (32 in) diameter, 2 by 1 cylinder	835	30
1 m cube	1 000	35
15 × 15 cm (6 in × 6 in) plate bearing	170	6
23 × 23 cm (9 in × 9 in) plate bearing	570	20
30 × 30 cm (12 in × 12 in) plate bearing	1 380	48
Deformability		
NX borehole jack	130	4.6
30 cm (12″) diameter, plate bearing	950	33.6
91 cm (36″) diameter, plate bearing	26 000	908
Pressure tunnel, 1.5 m diameter, 6 m long	82 000	1415
"Petite sismique"	Up to several thousands m³	

These volumes are to be compared to those of the following laboratory tests:

- — NX sample .... 244 cm³ (15 in³)
- — 10 cm (4 in) cube .... 1000 cm³ (64 in³)
- — 20 cm (8 in) diameter, 2 by 1 cylinder .... 0.013 m³ (800 in³)

Heuze, 1980

Summary of scale effects

Heuze, 1980

Name of project, date and reference	Rock type	Type of field test	No. of tests	E_F* (GPa)	E_L* (GPa)	E_F/E_L
Waldeck II 1973 (41)	Greywacke (S)	Plate bearing		5.0	20.0	0.25
		Tunnel relaxation		15.0		0.75
Mica Project 1974 (37)	Quartzite Gneiss (M) (M)	Plate bearing	12	27.6	27.0	1.04
		Flat jacks	19	28.8		1.07
		Goodman jack	132	16.6		0.61
Channel Tunnel 1975 (53)	Chalk (S)	Plate bearing		2.4	0.7	3.42
LG-2 Project 1976 (38)	Massive Granite (I)	Plate bearing		50.0	80.0	0.62
Dinorwic 1977 (19)	Slate (M)	RQD index		50.0	105.0	0.48
Elandsberg 1977 (8)	Greywake (S)	Plate bearing	33	39.6	73.4	0.54
		Small flat jacks	37	45.5		0.62
		Large flat jacks	3	42.2		0.57
		Goodman jack	39	28.4		0.39
		Tunnel relaxation	23	42.5		0.58
		Petite sismique	43	26.0		0.35
		RQD prediction	34	35.5		0.48
		RMR prediction	45	41.3		0.56
	Phyllite (M)	Small flat jack	9	33.7	56.0	0.60
		Goodman jack	6	12.0		0.21
		Tunnel relaxation	4	20.0		0.36
		Petite sismique	25	15.4		0.27
		RQD prediction	5	11.2		0.20
		RMR prediction	7	20.1		0.36
Orange River 1976 (8)	Dolerite (Verwoerd Dam) (I)	Plate bearing		25.5		0.36
		Pressure chamber		25.2		
		Tunnel relaxation		23.5		
		Petite Sismique		27.3		
	Shale (S)	Plate bearing		12.8		0.40
		Petite sismique		12.1		
	Dolerite (le Roux Dam) (I)	Plate bearing		26.0		0.30
		Pressure chamber		22.0		
		Tunnel relaxation		31.8		
		Petite sismique				
	Mudstone Siltstone Sandstone (S)	Plate bearing		13.0		0.70
				17.8		
		Pressure chamber		10.0		

Summary of scale effects (cont.)

Heuze, 1980

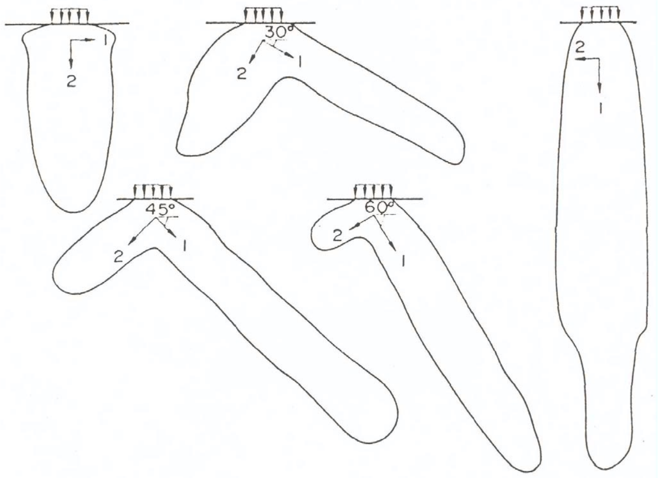
Ratios E_F/E_L for the Three Rock Classes

Rock class	No. of results	Mean	Std. Dev.
Igneous	15	0.35	0.16
Metamorphics	41	0.36	0.23
Sedimentaries	22	0.42	0.26

Ratios E_F/E_L for Various Types of Field Deformability Tests

Type of test	No. of results	Mean	Std. Dev.
Plate bearing	27	0.32	0.26
Full scale deformation	14	0.44	0.26
Flat jacks	10	0.54	0.27
Borehole jack or dilatometer	9	0.33	0.17
Pressure chamber	8	0.45	0.22
Petite sismique	5	0.34	0.05
Others	5	0.42	0.14

Plate tests on bedded (anisotropic) rocks



Pressure bulb shape under a plate, influenced by rock mass anisotropy (Singh, 1973a). In the figure, direction 1 is parallel to the bedding planes.

Plate tests on bedded rocks (cont.)

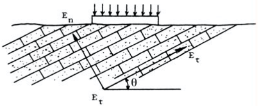


Plate shape and sensing points	E_n/E_t	Angle of anisotropy (θ)	$K = E_{calc}/E_r$ Flexible at center	Flexible at edge	Rigid	Effect of
1, 8	1	0	1.11	0.85	0.91	Plate geometry
2, 8	1	0	1.03	0.67	0.80	
2, 30°, 1, 8	1	0	0.94	0.98	0.87	
1, 8	1/3	0	1.67	1.32	1.41	Angle of anisotropy
same	1/3	30	1.28	1.15	1.12	
same	1/3	45	1.68	1.47	1.47	
same	1/3	60	2.33	1.95	2.06	

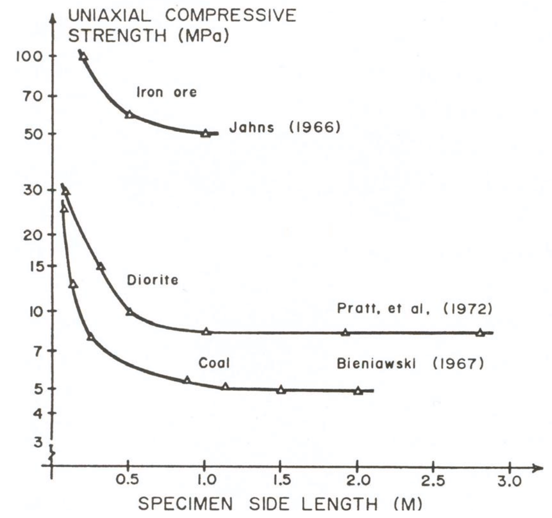
When conducting plate bearing tests on anisotropic rocks, the modulus calculated from an isotropic solution can be in error due to the rock mass anisotropy and possibly due to the plate geometry. Results based on 2-D finite element simulations (Heuze and Salem, 1977).

Strength of Rock Masses

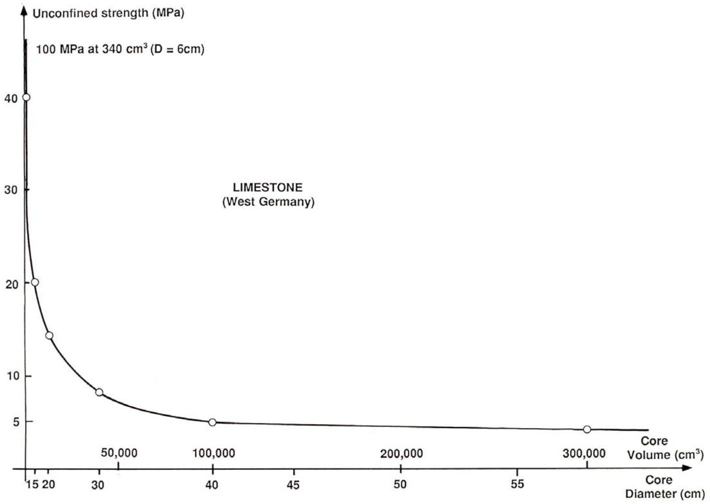
In-situ strength tests - Compressive strength

See Bieniawski and Van Herden, 1975

Compressive strength scale effects



Compressive strength scale effects (cont.)



The 1980 Hoek and Brown rock mass strength equation

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m\frac{\sigma_3'}{\sigma_{ci}} + s\right)^{0.5}$$

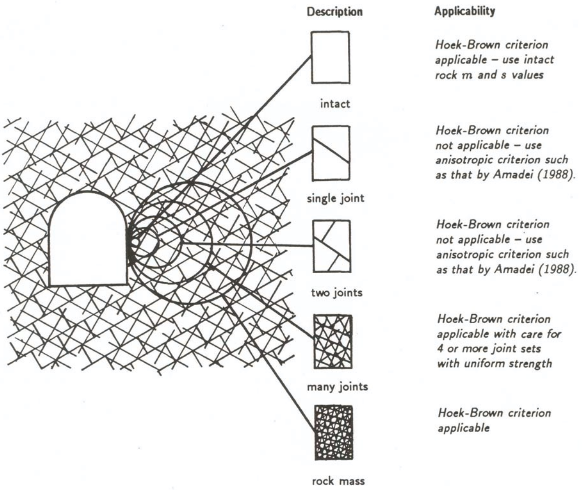
where σ_1' and σ_3' are the major and minor effective principal stresses at failure

σ_{ci} is the uniaxial compressive strength of the intact rock material and

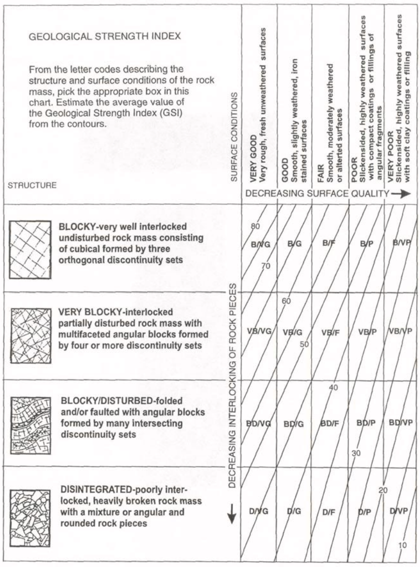
m and s are material constants, where $s = 1$ for intact rock.

The 1988 update

Hoek and Brown provided a figure indicating when to apply the rock mass criterion. Note the reference to Amadei, 1988.



The Geological Strength Index - GSI (Hoek,1994)



GSI and RMR (Hoek and Brown,1997)

- Hoek and Brown have stated that the GSI can be obtained from the RMR of Bieniawski (1989) as follows:

$$GSI = RMR_{89} - 5$$

where RMR_{89} has the groundwater rating set to 15 and the adjustment for joint orientation is set to zero.

- This correlation should not be used for poor quality rock masses, i.e. with GSI < 25.

The 2002 Update

The entire procedure is available online at: www.rocscience.com, in the program RockLab, that includes tables and charts to estimate σ_{ci}, m_i, and the GSI. The strength equations are:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

where m_b is a reduced value of the material constant m_i and is given by

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses.

The m_i coefficient

The mi coefficient should be determined by statistical analysis of the results of a set of triaxial tests. When that is not available, the table below can be used for estimates (Hoek and Brown, 1997).

Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates

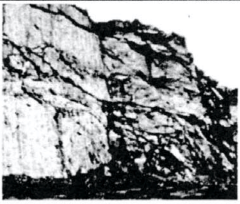
Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				—Greywacke— (18)		
	Non-Clastic	Organic		—Chalk— 7		
				—Coal— (8–21)		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non-foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25–31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists 4–8	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
			Diorite (28)		Andesite 19	
	Dark		Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

*These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

The 2002 Update - The Damage factor

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$

The 2002 Update - The Damage factor (cont.)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

Field modulus vs. the 2002 GSI

$$E_m(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{((GSI-10)/40)}$$

Other, proposed correlations

Empirical equation	Required parameters	Limitations	Equation
Bieniawski [2]	RMR	RMR > 50	$E_M = 2RMR - 100$
Serafim and Pereira [4]	RMR	RMR ⩽ 50	$E_M = 10^{(RMR-10)/40}$
Nicholson and Bieniawski [5]	E_i and RMR	—	$E_M = E_i[0.0028RMR^2 + 0.9\exp(RMR/22.82)]$
Mitri et al. [6]	E_i and RMR	—	$E_M = E_i[0.5(1 - (\cos(\pi * RMR/100)))]$
Hoek and Brown [7]	GSI and UCS	UCS ⩽ 100 MPa	$E_M = \sqrt{\frac{UCS}{100}}10^{(GSI-10)/40}$
Barton [1]	Q		$E_M = 10Q_c^{1/3}Q_c = Q\frac{UCS}{100}$
Palmström and Singh [8]	RMi	—	RMi > 0.1, $E_M = 5.6RMi^{0.375}$ 0.1 < RMi < 301, $E_M = 7RMi^{0.4}$
Kayabasi et al. [9]	E_i, RQD and WD	—	$E_M = 0.135\left[\frac{E_i(1 + RQD/100)}{WD}\right]^{1.1811}$

[1] I. J. Rock Mechanics, v.39, 185-216, 2002

[2] I. J. Rock Mechanics, v. 15, 237-247, 1978

[4] Proc. Symp. Eng. Geol. Underground Openings,Lisbon, 1983

[5] I. J. Mining and Geol. Eng., v. 8, 181-202, 1990

[6] SME Annual Mtg., Albuquerque, 94-116, 1994

[7] I. J. Rock Mechanics, v. 34, n. 8, 1165-1186, 1997. Note the 2002 update, on the previous slide.

[8] Tunneling and Undergr. Space Technology, v. 16, ,115-131, 2001

[9] I. J. Rock Mechanics, v.40, 55-63, 2003

After Gokceoglu et al, 2003

Additional strength criteria for rock masses

- **Coal seams (after Kalamaras and Bieniawski, 1993) :**

$$\sigma_1/\sigma_c = b\,[\sigma_3/\sigma_c]^{0.6} + a$$

$$b = \exp\,[(RMR+20)/52]$$

$$a = \exp\,[(RMR-100)/12]$$

- **Other criteria: see, for example, Sheorey, 1997**

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